

ADDRESSING CONSERVATISM AND UNCERTAINTY
ASSOCIATED WITH USACE AIRFIELD DESIGN CRITERIA

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Abstract

Next-generation USACE design methods for airfield pavements will provide solutions with accompanying reliability. Designs will be able to account for variability in traffic, structure, and materials. A necessary component will be accounting for conservativeness and uncertainty associated with failure criteria. This paper presents initial findings related these two characteristics for both the California bearing ratio (CBR) and layer elastic (LE) methods of design, with consideration only for subgrade failure. Based on a limited number of comparisons between predicted life and test section life, the CBR procedure does not impose embedded conservatism. The available data provided a means of assigning uncertainty to the CBR subgrade failure criteria. However, using the same set of test section data, the LE procedure was found to include various levels of embedded conservatism. Assigning uncertainty to the subgrade failure criteria was not possible and will require further study.

Introduction

Currently, the U.S. Army Corps of Engineers (USACE) has two design procedures for flexible pavements: the California bearing ratio (CBR) procedure and the layered elastic (LE) procedure. The CBR design procedure was originally developed in the state of California for the purpose of roadway design. The process of adapting the procedure to airfield design was started in the 1940's, with urgency necessitated by World War II. The procedure has evolved since that time as aircraft have become heavier and were equipped with multi-wheel gears and higher tire pressures. Ahlvin (1991) published a thorough description of the history of the CBR design procedure.

One of the objectives in USACE research in airfields and pavements is to develop pavement design methods that are more mechanistic than empirical in nature. The LE design procedure is one step in a continuing effort toward that objective. The LE design procedure was made possible by the development of software tools that could analyze the response of multilayered elastic structures to surface loads, such as CHEVRON (Michelow 1963) and BISAR (Peutz 1968). Early work in developing an LE procedure for the USACE was conducted by Barker and Brabston (1975).

Recent advances in USACE pavement design methods include the explicit consideration of uncertainty through Monte Carlo simulations. The simulations provide a means to account for variability in traffic, structural thicknesses, and material properties. They also provide a convenient means to account for changes over time for any of the design input variables. Design solutions and performance predictions can be expressed in terms of reliability. These reliability-based engineering solutions must account for both conservatism and uncertainty related to failure criteria. This paper describes some initial findings in this effort in relation to flexible pavements.

USACE CBR Design Procedure

A brief review of the USACE CBR thickness design procedures will facilitate further discussion of the paper topic. The principle of CBR design is to protect each layer of a pavement beneath the asphalt concrete with sufficient thickness so that the applied traffic loads do not

cause substantial permanent deformation. The most recent form of the design equation is referred to as the "cubic equation." For preservation of its original form, customary units are necessary for this description, but conversions from SI units are provided.

$$\frac{t}{a\sqrt{A}} = -0.0481 - 1.1562 \left[\text{Log} \frac{CBR}{Pe} \right] - 0.6414 \left[\text{Log} \frac{CBR}{Pe} \right]^2 - 0.4730 \left[\text{Log} \frac{CBR}{Pe} \right]^3 \quad (1)$$

where,

t = pavement thickness above subgrade, in. (1 in. = 0.0254 m)

a = load repetition factor,

A = contact area of all tires, in.² (1 in.² = 645 mm²)

CBR = California bearing ratio (%), and

Pe = equivalent single wheel load (ESWL), applied as a pressure, psi (1 psi = 6.895 kPa).

The equivalent single wheel load (ESWL) is a mathematical scheme developed to convert a multiple-wheel gear to a single-wheel gear that has similar characteristics; i.e., a single tire that represents an equivalent damaging effect to the pavement as the multiple-wheel gear. The converted gear can then be easily analyzed with the CBR equation. The ESWL is applied over an area equal to the contact area of one tire from the gear. The magnitude of pressure (Pe) associated with an ESWL is such that it causes a maximum deflection at the top of the subgrade equal to the maximum deflection caused by the entire gear. The calculation of Pe is achieved with the assumption that the pavement is composed of a single material that is linear elastic, homogeneous, and isotropic.

The load repetition factor (α) was added to the CBR equation so that the design procedure could be applied to a wide range of traffic levels and gear configurations, i.e. gear assemblies with as many as 12 tires. Figure 1 shows α as a function of both aircraft coverages and the number of wheels used to compute ESWL. Aircraft "coverages" are similar to passes, but have the added complexity of accounting for aircraft wander patterns and footprints. Producing these α curves involved much engineering judgment because the available data were scarce and applied primarily to aircraft coverage levels of less than 5000 (Cooksey and Ladd 1971).

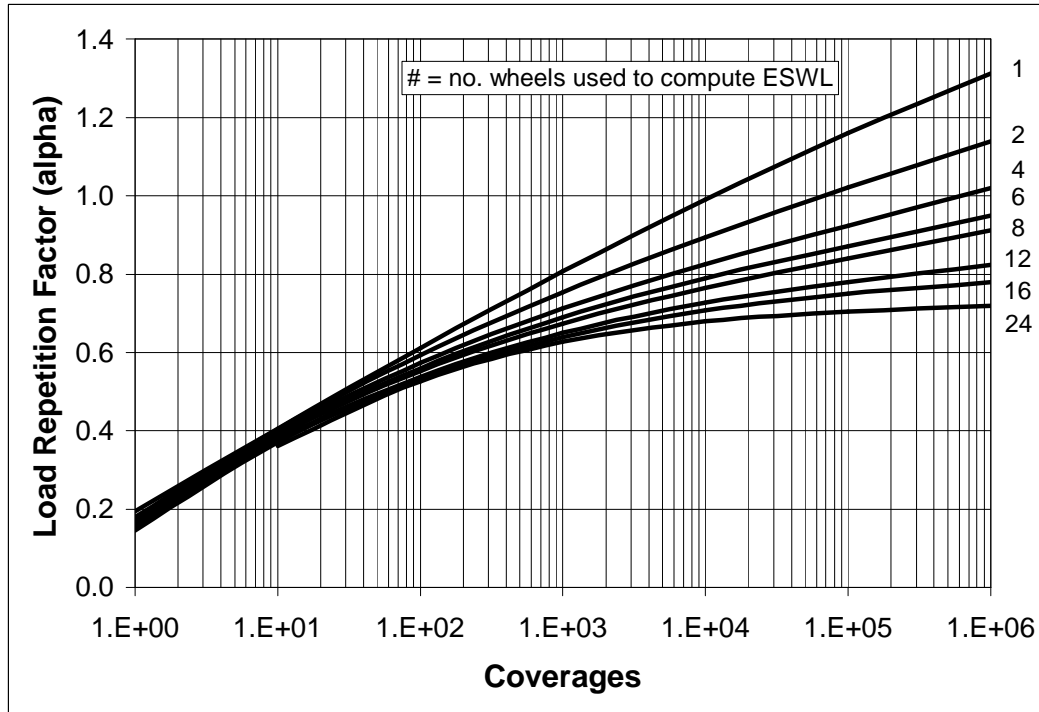


Figure 1. Load Repetition Factor (**a**)

USACE Layered Elastic (LE) Design Procedure

The current USACE LE procedure uses the WESLEA analysis software, which is an adaptation of layered elastic analysis software originally developed by Jacob Uzan, Technion - Israel Institute of Technology. The capabilities and assumptions associated with using this program are similar to many other programs used for LE design (Chou 1992 and USACE 2001).

- a. The pavement is a multilayered structure and the material in each layer is assumed to be weightless, homogeneous, isotropic, and linearly elastic.
- b. Relevant material properties for each layer include an elastic modulus and a Poisson ratio.
- c. All layers have infinite horizontal dimensions and the lowest layer has an infinite vertical dimension.
- d. Adjacent layers remain in contact, but each interface can be assigned various degrees of bond ranging from complete (no relative horizontal displacements) to none (independent horizontal displacements). All applications of the program in this paper assume complete bond.
- e. Applied loads are static, circular, and have uniform contact stress. Multiple loads are analyzed by adding their individual effects, which are assumed to be independent.

The principle of the LE design procedure is to prevent failure of pavements by either excessive traffic-induced cracking in the asphalt concrete surface layer, excessive traffic-induced cracking in a chemically stabilized base/subbase, or excessive shear deformation in the subgrade soil. Potential for cracking is calculated as a function of elastic tensile strains at the bottom of

either the asphalt surface layer or the chemically stabilized layer. Potential for subgrade shear is calculated as a function of elastic compressive strains at the top of the subgrade soil layer. The effects of mixed traffic are added by means of a damage factor (DF) where the cumulative effect of all aircraft equals the sum of all DFs, one calculated for each aircraft.

$$DF = \frac{n}{N} \quad (2)$$

where,

n = number of effective strain repetitions

N = number of allowable strain repetitions (prior to failure)

Because this paper is concerned with comparing conservatism of CBR design to LE design, the subgrade strain criteria are of primary interest. These criteria have evolved from those initially proposed by Barker and Brabston (1975), which were developed through a combination of test section analyses and engineering judgment. The criteria are encompassed in the following equation, which is also shown in Figure 2.

$$\text{allowable repetitions} = 10,000 \cdot \left(\frac{A}{S_V} \right)^B \quad (3)$$

where,

$$A = 0.000247 + 0.000245 \log(M_R)$$

M_R = resilient modulus of the subgrade, psi (1 psi = 0.006895 MPa)

S_V = vertical strain at the top of the subgrade (m/m)

$$B = 0.0658 (M_R)^{0.559}$$

The resilient modulus (M_R) of subgrade soil would most preferably be determined through repetitive laboratory testing, as described in Appendix L of the USACE design manual (USACE 2001). This procedure is an abbreviated version of that prescribed by the American Association of State Highway and Transportation Officials (AASHTO 2000). For each aircraft, the appropriate strain to be used in the equation above is the maximum vertical strain at the subgrade surface.

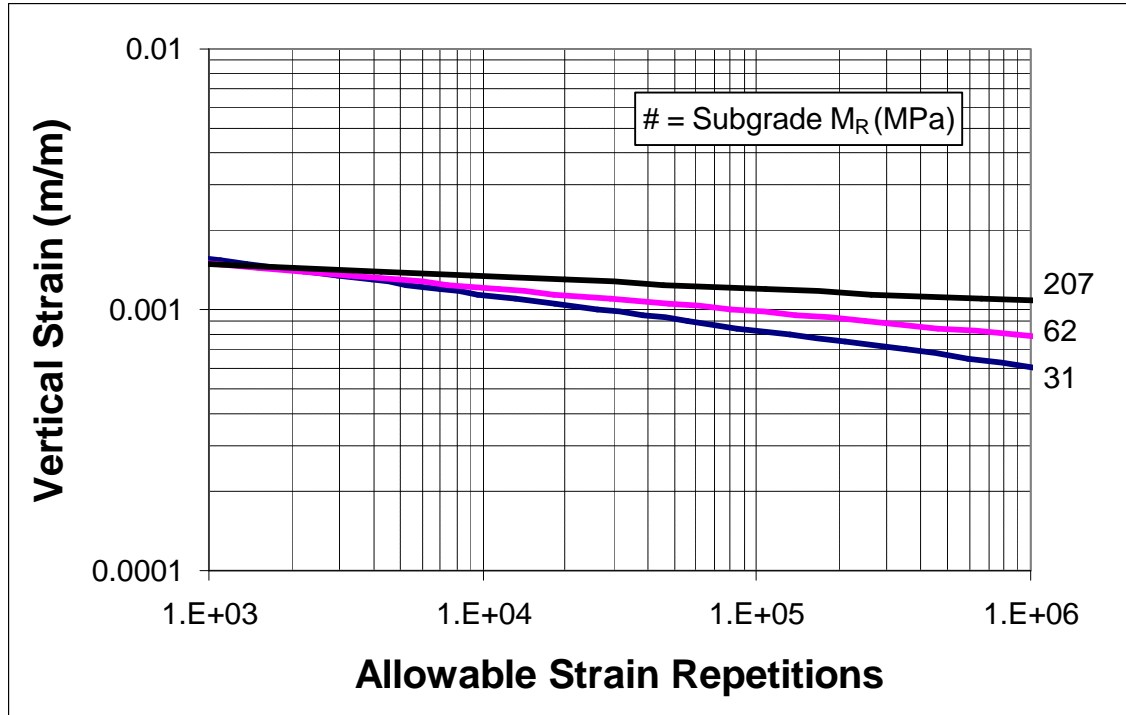


Figure 2. Subgrade Strain Criteria for the USACE Layered Elastic Design Method (after USACE 2001)

Counting the number of strain repetitions with aircraft passes requires consideration for both aircraft wander and the effects of multiple tires. The LE design procedure accounts for the effects of tandem tires on strain repetitions by assuming that the zone of influence for each tire follows a 2:1 (vertical:horizontal) slope through unbound materials and follows a 1:1 (vertical:horizontal) slope through bound materials. This differentiation between bound and unbound materials gives credit to the bound materials for their greater load-spreading capabilities. If the tandem distance is relatively large or the subgrade surface relatively shallow, then the subgrade will experience two separate strain pulses. If the tandem distance is relatively small or the subgrade surface relatively deep, then the subgrade will experience a single strain pulse. Cases falling between these two extremes are computed using linear interpolation. The calculations for strain repetitions are facilitated by an “effective thickness” (T_e) concept for pavement above subgrade. Effective thickness is defined as:

$$T_e = 2 \cdot (\text{thickness of bound materials}) + 1 \cdot (\text{thickness of unbound materials}). \quad (4)$$

Then, using variables as defined in Figure 3, strain repetitions per aircraft pass are calculated as follows.

- If $T_e \leq (TS - TL)$, or zone A in Figure 3, then strain repetitions per aircraft pass = 2.
- If $T_e \geq 2(TS - TL)$, or zone C in Figure 3, then strain repetitions per aircraft pass = 1.
- Else, zone B in Figure 3, and

$$\text{strain repetitions per aircraft pass} = \frac{3 \cdot (TS - TL) - T_e}{TS - TL} \quad (5)$$

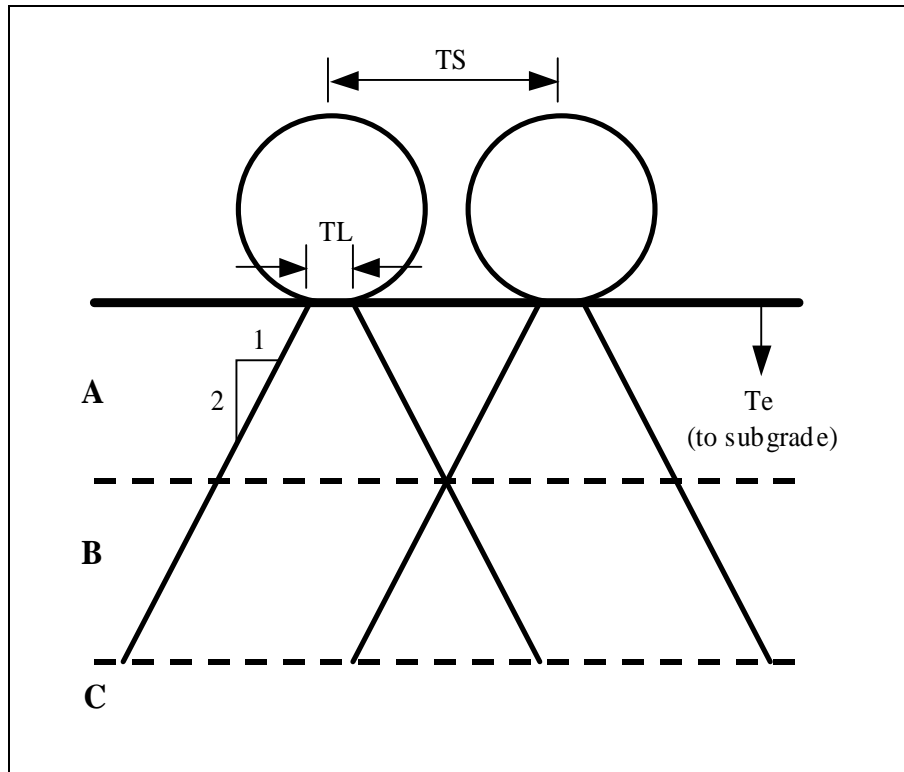


Figure 3. Assumptions for Calculating Number of Strain Repetitions Per Aircraft Pass

The combined effect of twin tires can best be explained after first introducing aircraft wander assumptions. The LE design procedure assumes that aircraft wander patterns are normally distributed. Based on field measurements (Brown and Thompson 1973), the distribution of aircraft on taxiways is assumed to have a standard deviation of 0.77 m and the distribution of aircraft on runways is assumed to have a standard deviation of 1.55 m.

Considering a point at a subgrade surface, with equivalent pavement thickness above equal to “ T_e ” (see Figure 4), the aircraft passes that influence this particular point are assumed to include those passes for which any part of either tire touches any point of the pavement surface that is bounded by the 2:1 (vertical:horizontal) slopes projecting from the point in question. The percent of aircraft operations that apply strain to this particular point on the subgrade surface would

equal one-half the area between the vertical dashed lines, accounting for area under each of the two probability distributions.

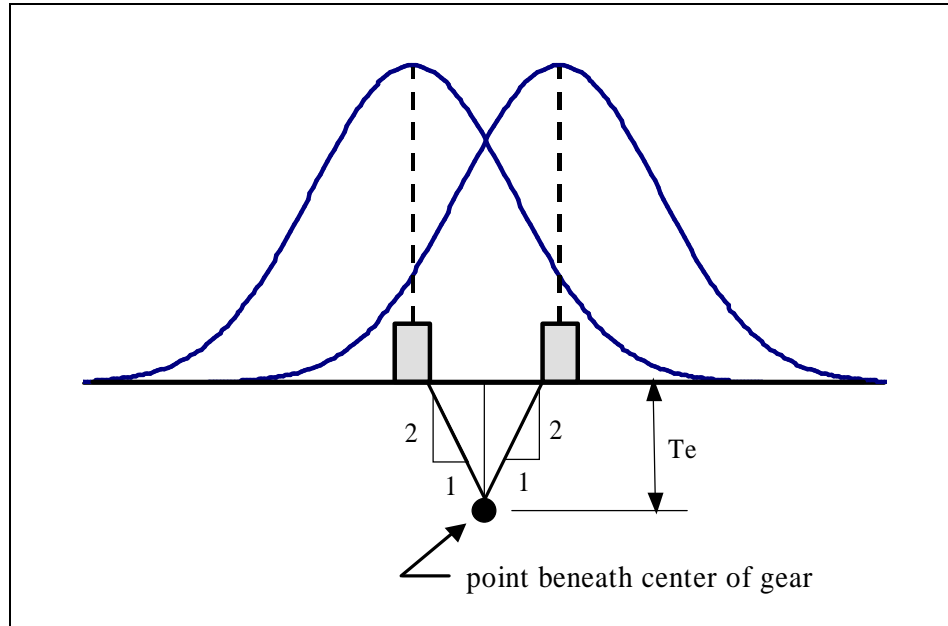


Figure 4. Accounting for the Influence of Twin Tires and Aircraft Wander (Point at Depth of 1.0 m)

Conservativeness and Uncertainty for the CBR Failure Criteria

The CBR design procedure was developed to provide 50 percent reliability; conservatism was not built into the design procedure. Potter (1985) confirmed this proposition by investigating conservatism in the CBR procedure after inclusion of the load repetition factor (α). Potter compared the performance of 28 historical test sections to the cubic design equation. For the purposes of review and elaboration in this paper, three items were removed from Potter's list. Two items were removed because they represented field failures, so the estimates for traffic were not as accurate as that commonly attained by accelerated pavement testing. The third item was removed because the wearing surface consisted only of a seal coat over crushed aggregate base course. The remaining 25 test items are summarized in Table 1. The traffic on all the remaining test items had a minimum individual tire load of 133 kN (30,000 lb), tire contact areas from 0.059 to 0.968 m² (91 to 1500 in.²), subgrade CBR values from 3.7 percent to 20 percent, and pavement thicknesses above subgrade from 0.254 to 1.24 m (10 to 49 in.).

The performances of test sections are compared to the USACE subgrade failure criteria for CBR design in Figure 5. In order to be consistent with the original form of the equation, points are plotted using customary units. The figure shows that the performance of these test sections is evenly distributed on both the under-design and over-design sides of the design equation. Also, a confidence interval can be used to reflect criteria uncertainty, as shown in Figure 6.

Table 1. Selected Pavement Test Sections (after Potter 1985)

Point No.	Type of Gear	Gear Load (kN)	Tire Contact Area (m ²)	CBR (%)	Thickness Above Subgrade (m)	Coverage s to Structural Failure ^a	Pass-to-Coverage Ratio
1	single tire	133	0.097	14.0	0.305	216	6
2	single tire	133	0.097	7.0	0.305	178	6
3	single tire	133	0.097	6.0	0.305	203	6
4	single tire	133	0.184	3.7	0.381	120	5
5	single tire	222	0.184	3.7	0.381	6	5
6	single tire	222	0.184	4.4	0.610	200	5
7	single tire	890	0.968	6.0	0.991	150	4
8	single tire	890	0.968	9.0	0.110	1700	4
9	single tire	890	0.968	16.0	0.457	10	4
10	single tire	890	0.968	18.0	0.521	60	4
11	single tire	890	0.968	15.5	0.597	360	4
12	single tire	890	0.968	17.5	0.762	1500	4
13	single tire	890	0.968	8.0	0.124	1300	4
14	twin	311	0.213	20.0	0.254	2000	4
15	twin tandem	534	0.097	12.0	0.406	310	1.5
16	twin tandem	534	0.097	5.0	0.406	90	1.5
17	twin tandem	534	0.097	15.0	0.406	1500	1.5
18	twin tandem	667	0.169	16.0	0.356	1000	2
19	twin tandem	1070	0.187	3.8	0.838	40	1.5
20	twin tandem	1070	0.187	4.0	0.838	40	1.5
21	twin tandem	1070	0.187	4.0	1.07	280	1.5
22	12-tire gear	1600	0.184	3.7	0.381	8	0.69
23	12-tire gear	1600	0.184	4.4	0.610	104	0.69
24	12-tire gear	1600	0.184	3.8	0.838	1500	0.69
25	12-tire gear	1600	0.184	4.0	0.838	1500	0.69

^a number of coverages = maximum number of times any point on the pavement surface has been “covered” by a tire during trafficking. References: Points 1 through 3 and 15 through 17 (USACE 1950), Points 4 through 6 and 19 through 25 (Ahlin et al 1971), Points 7 through 13 (O.J. Porter & Co. 1949), Points 14 and 18 (USACE 1952).

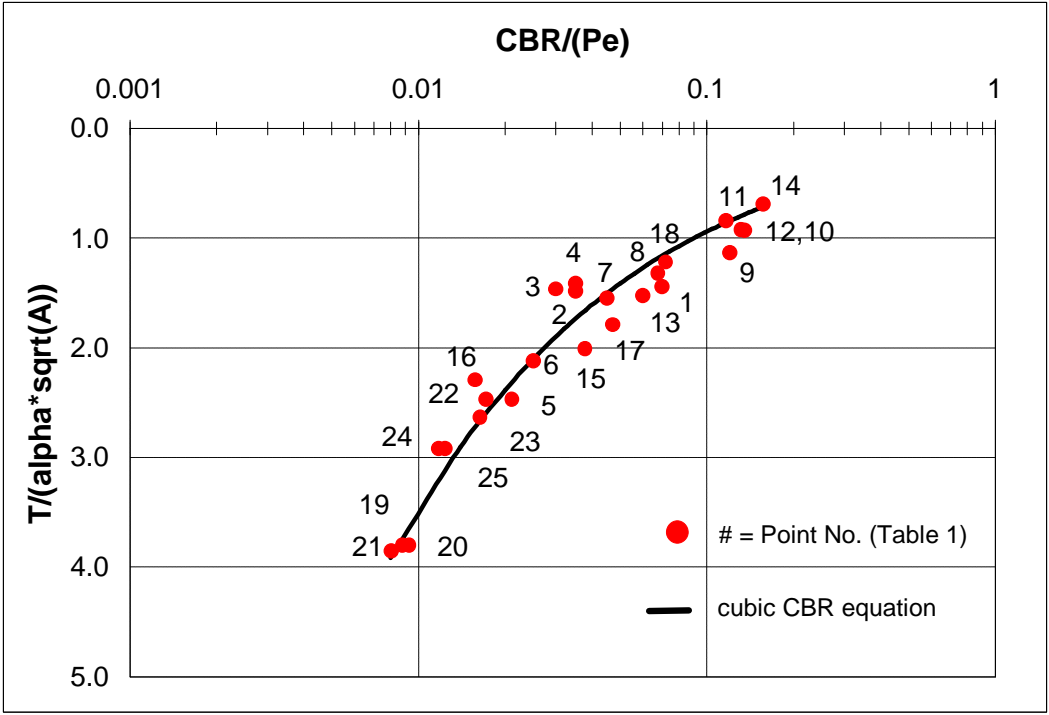


Figure 5. Comparison between Historical Test Section Performance and the Cubic CBR Design Equation (after Potter 1985)

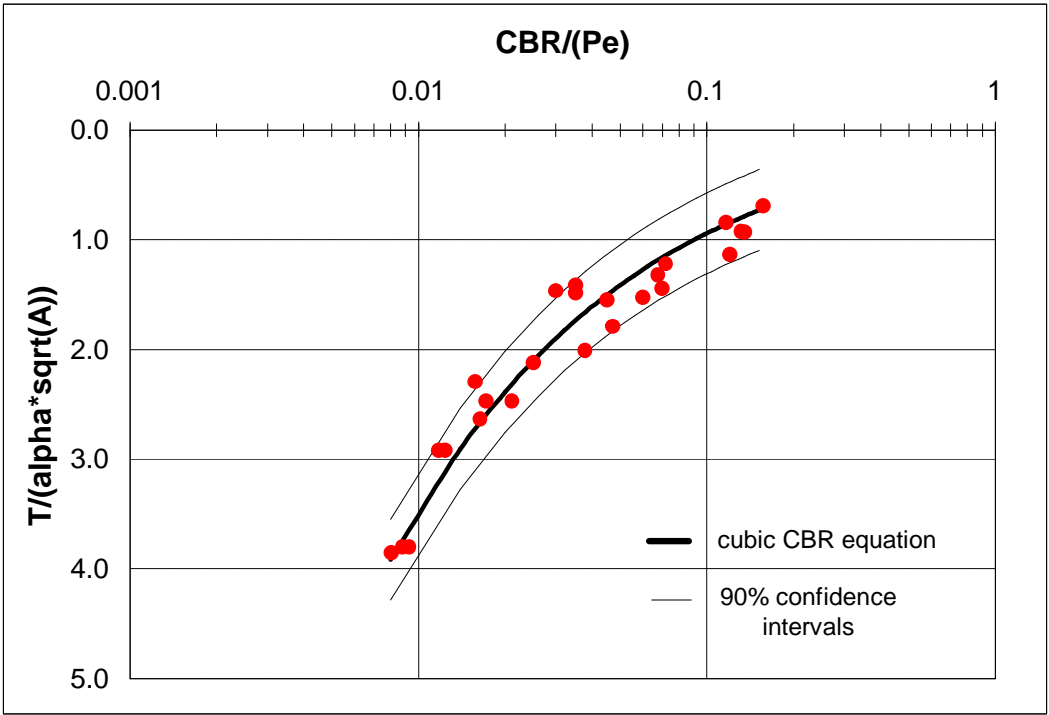


Figure 6. Cubic CBR Equation with an Associated Confidence Interval

Conservativeness and Uncertainty for the LE Failure Criteria

The intent of USACE LE design was to provide 50 percent reliability, similar to that provided by the CBR design. However, comprehensive documentation for reliability associated with USACE LE design could not be found in literature. During the early development of the LE design procedure, Barker and Brabston (1975) made a limited number of comparisons between test section performance and the subgrade strain criteria. They used 15 pavement test sections, including nine pavements with unbound bases and six pavements with stabilized bases. All pavements were constructed on CH subgrade with CBR equal to 4 percent. With a single plot comparing the developed criteria to test section performance, Barker and Brabston show the criteria to be conservative. They properly qualified these comparisons with the statement that, “Although the limited comparisons presented do not represent a complete verification of the subgrade strain criteria, the comparisons do lend credibility to the criteria.”

For purposes of this study, the conservativeness of the LE design procedure was analyzed using the same list of test sections as those presented earlier for analyzing the CBR design procedure. Because these test sections were trafficked prior to the rise in popularity of either resilient modulus tests or back-calculating pavement layer moduli, several assumptions were necessary. The assumptions that follow are among those recommended to USACE pavement engineers by the manual “Pavement Design for Airfields” (USACE 2001) when resilient modulus laboratory tests are not available. Similarities and differences between the following assumptions and those used by Barker and Brabston (1975) during the development of the layered elastic design method will be identified subsequently.

For the purposes of this paper, the elastic moduli of subgrade materials were estimated according to (Heukelom and Klomp 1962):

$$E(MPa) = 10.34 \cdot CBR(\%) \quad (6)$$

This relationship has been found to be reasonable as long as the CBR of the soil is less than about 20 percent (Huang 1993).

The elastic moduli of granular materials were estimated as a function of both layer thickness and modulus of the underlying layer. Izatt, Lettier, and Taylor (1967) originally developed these rules-of-thumb for roadway pavements.

$$\begin{aligned} \text{For subbase : } E_n &= E_{n+1} \cdot [1 + 7.18 \cdot \text{Log}(t) - 1.56 \cdot \text{Log}(E_{n+1}) \cdot \text{Log}(t)] \\ \text{For base : } E_n &= E_{n+1} \cdot [1 + 10.52 \cdot \text{Log}(t) - 2.10 \cdot \text{Log}(E_{n+1}) \cdot \text{Log}(t)] \end{aligned} \quad (7)$$

where,

E_n = modulus of layer in question, psi (1 psi = 0.006895 Mpa)

E_{n+1} = modulus of the underlying layer, psi (1 psi = 0.006895 Mpa)

T = thickness of underlying layer, in. (1 in. = 25.4 mm)

In order to use these equations with relatively thick airfield pavements, the USACE has added rules for subdividing pavement layers (USACE 2001). When a subbase layer is thicker than 0.20 m (8 in.), it is divided into sublayers of equal thickness, each less than 0.20 m (8 in.). When a base layer is thicker than 0.25 m (10 in.), it is divided into sublayers of equal thickness, each less than or equal to 0.25 m (10 in.).

Estimates for asphalt concrete moduli were obtained using published relationships between modulus, temperature, and loading frequency (Kingham and Kallis 1972). Pavement temperature estimates were either available from test section reports or were estimated from reported air temperatures. Loading frequency was assumed as 2 Hz and a minimum modulus was established as 1380 MPa (200,000 psi). Because this paper is concerned with subgrade design criteria only, and because the asphalt layers in the test sections were not excessively thick, potential errors caused by inaccurate modeling of asphalt moduli were deemed tolerable.

The traffic was applied to these test sections with manually-driven load carts. Incremental lateral movements, made possible by markings on the pavement, simulated traffic wander. Resulting wander patterns were either discrete uniform distributions or discrete bell-shaped distributions. At the time of writing this paper, the LE software assumed distributions to be continuous and normal. Therefore, each applied wander pattern was best matched with a normal distribution that had an equivalent standard deviation to the applied traffic.

Input required for the LE analyses are shown in Table 2. Sublayer thicknesses and moduli for base and subbase layers are not shown in the table, but they were calculated according to the rules described above. Poisson ratios were assumed as 0.40 for subgrade, 0.30 for subbase and base, and 0.50 for asphalt concrete. Comparisons between predicted passes to failure and measured passes to failure are shown in Figure 7. Almost all the plotted points reside on the conservative side of the line of equality. Based on this set of data, it is apparent that conservatism is “built into” the LE design procedure. Also, the assignment of a confidence interval to reflect criteria uncertainty seems to be more complicated in this case than it was in the case of CBR design.

Further Discussion

The assumptions used by Barker and Brabston (1975), during the development of the USACE LE design procedure, were slightly different than those used in this paper. These differences are not viewed as significant to the findings presented, but they are listed below for completeness.

- a. Barker and Brabston (1975) estimated subgrade modulus by seeking a predetermined relationship between modulus and calculated deviator stress at the top of the subgrade. All subgrade soils in their analyses were CH with CBR of approximately 4 percent. The resulting average and standard deviation for estimated subgrade moduli were 31.4 MPa and 7.0 MPa, respectively.

b. Barker and Brabston (1975) assumed asphalt concrete modulus to be 1380 Mpa (200,000 psi) for all test sections because they were all trafficked in warm weather (during summer months in Vicksburg, MS).

c. To simplify computations, Barker and Brabston (1975) assumed each pass of each type of load cart applied one repetition of maximum strain to the same point on the surface of the subgrade. With respect to accounting for wander and multiple wheels, the assumptions used for analyses in this paper conformed to the current USACE LE design procedures, as presented earlier.

A significant difference between the current USACE LE design procedures and the original work by Barker and Brabston (1975) also deserves mention. While Barker and Brabston (1975) developed the subgrade strain criteria for 20-year pavement designs with annual strain repetitions between 1200 and 25,000, current USACE LE design procedures do not restrict the use of LE criteria to any particular range of passes. Thus, in some cases, current procedures inherently extrapolate the original work by Barker and Brabston (1975).

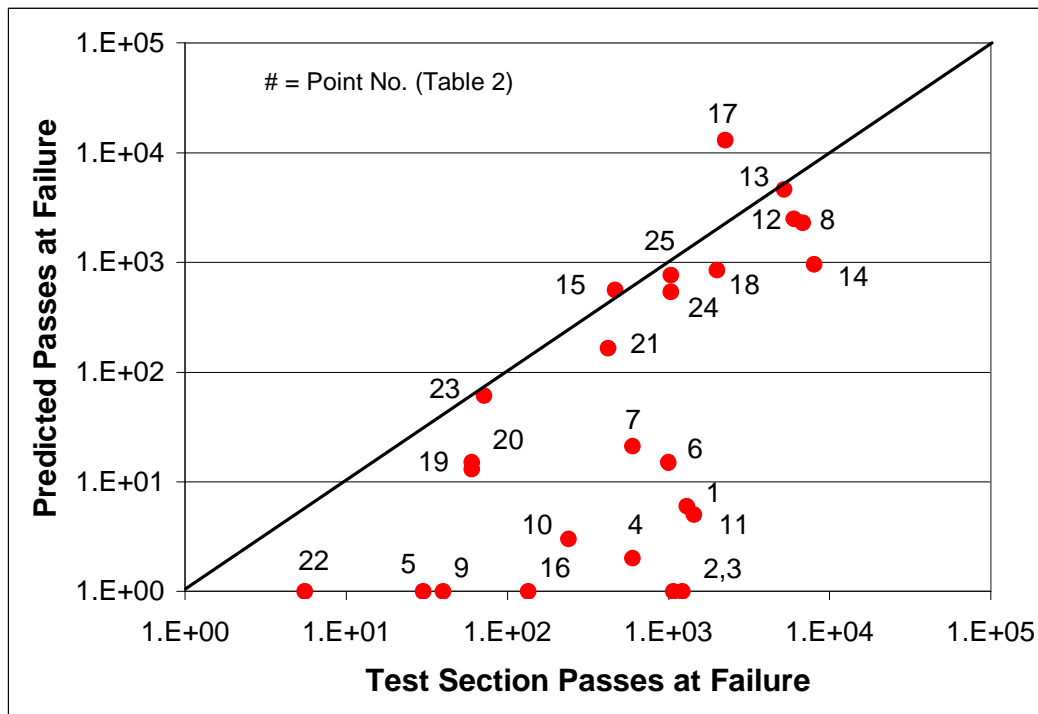


Figure 7. Comparison between Historical Test Section Performance and the LE Design Procedure

Table 2. Test Section Characteristics for LE Analysis

Point No.	Subgrade Modulus (MPa)	Subbase Thickness (m)	Base Thickness (m)	Asphalt Thickness (m)	Asphalt Modulus (MPa)	Tire Print Area (m ²)	Traffic Wander ^a (m)
1	145	n/a	0.254	0.0508	1380	0.097	0.43
2	72.4	n/a	0.133	0.0381	1380	0.097	0.43
3	62.1	n/a	0.254	0.0508	1380	0.097	0.43
4	38.3	0.152	0.152	0.0762	1380	0.184	0.55
5	38.3	0.152	0.152	0.0762	1380	0.184	0.55
6	45.5	0.191	0.152	0.0762	1380	0.184	0.55
7	62.1	0.161	0.165	0.1778	2480	0.968	0.92
8	93.1	0.182	0.191	0.1778	2480	0.968	0.92
9	166	n/a	0.153	0.1524	2480	0.968	0.92
10	186	n/a	0.178	0.1651	2480	0.968	0.92
11	160	0.064	0.184	0.1651	2480	0.968	0.92
12	181	0.115	0.178	0.1778	2480	0.968	0.92
13	82.7	0.191	0.165	0.1524	2480	0.968	0.92
14	207	n/a	0.178	0.0762	1380	0.213	0.94
15	124	n/a	0.184	0.0381	1380	0.097	0.23
16	51.7	n/a	0.184	0.0381	1380	0.097	0.23
17	155	n/a	0.184	0.0381	1380	0.097	0.23
18	166	n/a	0.14	0.0762	1860	0.169	0.80
19	39.3	0.153	0.152	0.0762	1860	0.187	0.39
20	41.4	0.153	0.152	0.0762	1860	0.187	0.39
21	41.4	0.168	0.152	0.0762	1860	0.187	0.39
22	38.3	0.152	0.152	0.0762	1380	0.184	0.57
23	45.5	0.191	0.152	0.0762	1380	0.184	0.57
24	39.3	0.153	0.152	0.0762	1380	0.184	0.57
25	41.4	0.153	0.152	0.0762	1380	0.184	0.57

^a in terms of standard deviation

References: Points 1 through 3 and 15 through 17 (USACE 1950), Points 4 through 6 and 19 through 25 (Ahlvin et al 1971), Points 7 through 13 (O.J. Porter & Co. 1949), Points 14 and 18 (USACE 1952).

Summary and Conclusion

Among the USACE design procedures for asphalt-surfaced airfield pavements, the California bearing ratio (CBR) procedure has been shown to offer a reliability of 50 percent. Absence of embedded conservatism permits the simple use of confidence intervals for quantifying the design uncertainty that is attributable to the pavement performance criteria. The USACE layered elastic (LE) design procedure has been shown to contain various levels of embedded conservatism, at

least for low levels of traffic. Until the conservatism is effectively quantified, accounting for the associated uncertainty will remain difficult.

Pavement engineering continues to move toward reliability-based design procedures. These approaches commonly consider the variability associated with construction materials, pavement thicknesses, and traffic. However, uncertainty associated with pavement performance criteria are considered less often. The contribution from this aspect of design to the reliability approach can be important and warrants consideration.

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